



UNIVERSITY OF RUHUNA

Faculty of Engineering

End-Semester 7 Examination in Engineering: March 2022

Module Number: CE 7251

Module Name: Coastal Engineering-Applications and Management (N/C)

[Three Hours]

[Answer all questions, each question carries TWELVE marks]

- Q1 (a) Use principles of physics to explain how (i) refraction and shoaling (ii) diffraction of water waves occur using sketches if necessary. Explain how the knowledge of above mentioned coastal processes can be employed in planning and design of harbour basins. [6 marks]
(b) Explain why wave run-up level on a sloping concrete seawall could be higher than on a rubble mound seawall having the same slope? [2 marks]
(c) It is planned to dump rocks around a monopile driven to seabed to arrest the scour. If the first (threshold) movement of a rock particle occurs when Shields critical shear stress, theta_c = 0.06 and bottom orbital velocity (U) is 1 m/s, calculate the (i) nominal rock diameter (d) and (ii) mass of rock to be used in the scour protection apron.

theta_c = tau / ((rho_s - rho)gd) ----- Eq. 1.1

tau = 1/2 * f_w * rho * U^2 ----- Eq. 1.2

Here, wave friction factor, f_w=0.2, rho_s=2650kg/m^3, rho=1030kg/m^3.

[4 marks]

- Q2 (a) Figure Q2 shows a sectional view of a caisson breakwater supported on a rock (rubble) foundation. The design wave height H=3m and wave period T=8 sec. Water depth, h_s=h_b=10m. Height of the caisson, h_w=6m. The caisson structure projects 1m above the still water level (h_c=1m). The water depth in front of the Caisson, d=5m. Calculate dynamic wave pressure on the seaward face of the caisson p_1, p_2, p_3 and uplift pressure (p_u) using relationships given below. Assume the caisson width is 3m and length is 6m. Angle of wave incidence is zero (beta = 0). Wave length, L refers to the water depth of 10m. Wave Table is in the Appendix.

eta* = 0.75 (1 + cos beta)H ----- Eq. 2.1

p_1 = 0.5 (1 + cos beta) (alpha_1 + alpha_2 cos^2 beta) rho_w g H ----- Eq. 2.2

p_3 = alpha_3 p_1 ----- Eq. 2.3

$$p_u = 0.5 (1 + \cos \beta) \alpha_1 \alpha_3 \rho_w g H \text{----- Eq. 2.4}$$

$$\alpha_1 = 0.6 + 0.5 \left[\frac{4\pi h_s/L}{\sinh(4\pi h_s/L)} \right]^2 \text{----- Eq. 2.5}$$

$$\alpha_2 = \text{smallest of } \frac{h_b - d}{3h_b} \left(\frac{H}{d} \right)^2 \text{ and } \frac{2d}{H} \text{----- Eq. 2.6}$$

$$\alpha_3 = 1 - \left(\frac{h_w - h_c}{h_s} \right) \left[1 - \frac{1}{\cosh(2\pi h_s/L)} \right] \text{----- Eq. 2.7}$$

[6 marks]

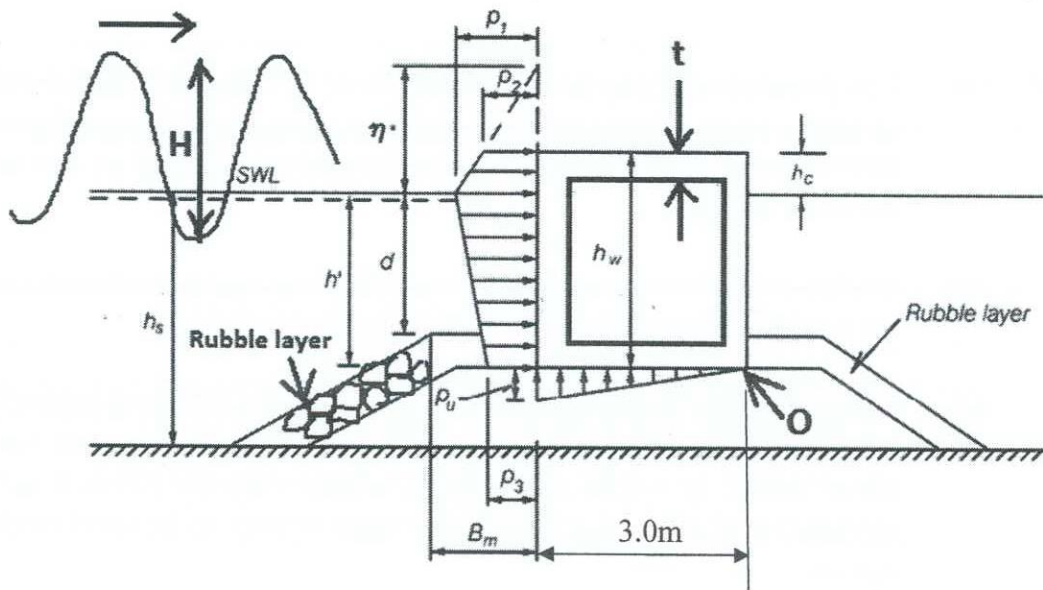


Figure Q2. Cross-section of a caisson breakwater

- (b) Calculate the total horizontal (F_H) and uplift (F_U) forces due to dynamic wave pressure on the caisson and the position of their line of action. By considering moments around point "O" calculate the thickness of the caisson box (t). Assume the caisson box is empty and density of concrete is 2400 kg/m^3 . [6 marks]

Q3 (a) (i) What are the assumptions made in deriving linear wave theory ?. (ii) Explain why linear wave theory is not valid in shallow water depths. [3 marks]

- (b) Explain how wave induced drag and inertia forces on a vertical mono-pile driven into the seabed are generated using sketches if necessary. Pressure distribution around the pile and displaced volume of water may be considered. [3 marks]

- (c) A cylindrical mono-pile with diameter $D=3\text{m}$ is to be driven into a sandy seabed in water depth $h=10\text{m}$ to construct a jetty exposed to waves. The design wave height, $H=2.5\text{m}$ and wave period is $T=8 \text{ sec}$. See Figure Q3 and use Wave Table given in the Appendix.

Linearised form of the velocity potential of a surface gravity wave is given by:

$$\phi = \frac{gH}{2\omega} \frac{\cosh k(z+h)}{\cosh(kh)} \sin(kx - \omega t) \text{-----Eq. 3.1}$$

(i) Derive horizontal orbital velocity (U) and acceleration (a_x) at the mid-depth. [3 marks]

(ii) Calculate **maximum (peak)** drag and inertia forces on the pile and estimate the total force. Assume velocity and acceleration at the **mid-depth** derived in (i) above, are applicable to the full submerged part of the pile. [3 marks]

$$\text{Drag force} = \frac{1}{2} \rho C_D A U^2 \text{-----Eq. 3.2}$$

$$\text{Inertia force} = \rho C_M \nabla a_x \text{-----Eq. 3.3}$$

Where, A = cross sectional area = $D \times h$ and the displaced volume of water, $\nabla = \frac{\pi D^2}{4} h$,
 $C_D=0.7$, $C_M=2.0$, density of water = 1030 kg/m^3 .

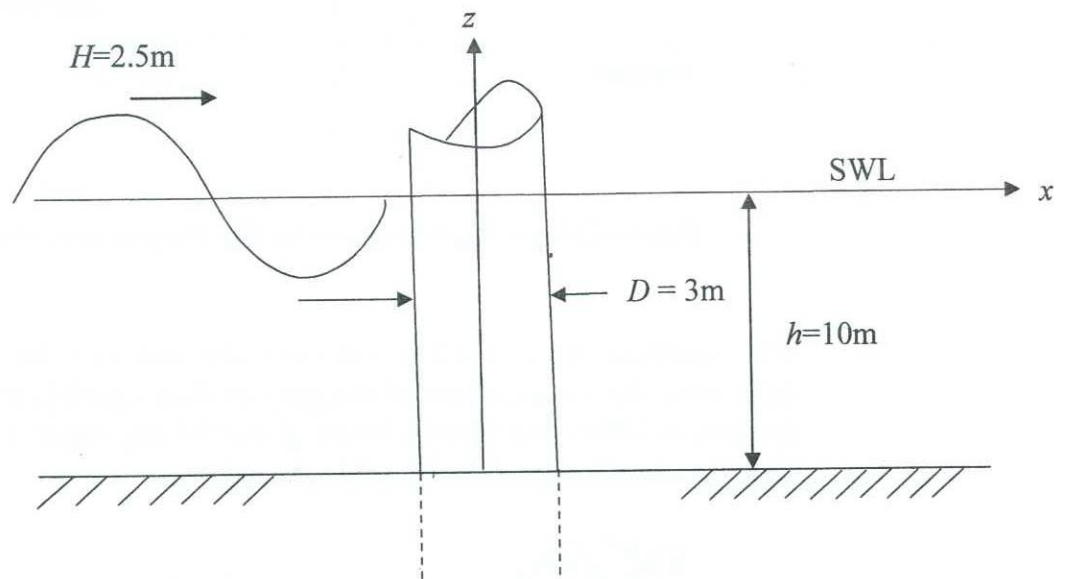


Figure Q3. Mono-pile

- Q4. (a) Time evolution of the beach planform up-drift of an impermeable groyne is given by the following equation 4.1 [See Figure Q4(a)].

$$y = 2\sqrt{Gt} \tan \alpha_b \left\{ \frac{1}{\sqrt{\pi}} \exp \left[-\left(\frac{x}{2\sqrt{Gt}} \right)^2 \right] - \frac{x}{2\sqrt{Gt}} \operatorname{erfc} \left(\frac{x}{2\sqrt{Gt}} \right) \right\} \quad \text{-----Eq. 4.1}$$

Calculate time, t_f , for the shoreline position to reach the tip of the groyne due to sand accretion.

[6 marks]

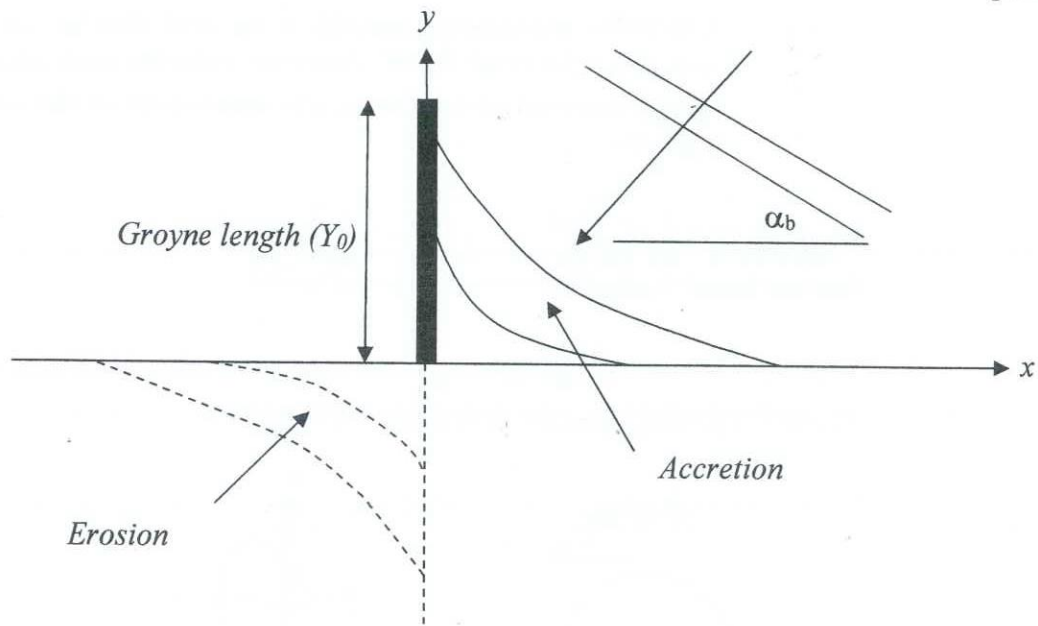


Figure Q4 (a). Sketch showing the groyne and wave approach.

- (b) Use equations 4.1 and 4.2 to calculate the distance the shoreline will extend 100 days after the construction of the groyne 50m updrift ($x=50\text{m}$), if the length of the groyne, is 150m and waves break at $\alpha_b=10$ deg angle to the shoreline. Sediment diffusion coefficient, $G(\text{m}^2/\text{sec})$ is given by:

$$G = \frac{KH_b^{5/2} \sqrt{g/\gamma_b}}{8 \frac{(\rho_s - \rho)}{\rho} \lambda (h_* + B)} \quad \text{----- Eq. 4.2}$$

$K=0.7$, breaker index, $\gamma_b = 0.78$, $\lambda = 1 - \text{porosity} = 0.6$, B (dry elevation above SWL)=3m, h_* (closure depth)=4m, h_b (depth of wave breaking)=1m, $\rho_s=2650\text{kg}/\text{m}^3$, $\rho=1030\text{kg}/\text{m}^3$. Use Figure Q4(b) to calculate complementary error function value.

[6 marks]

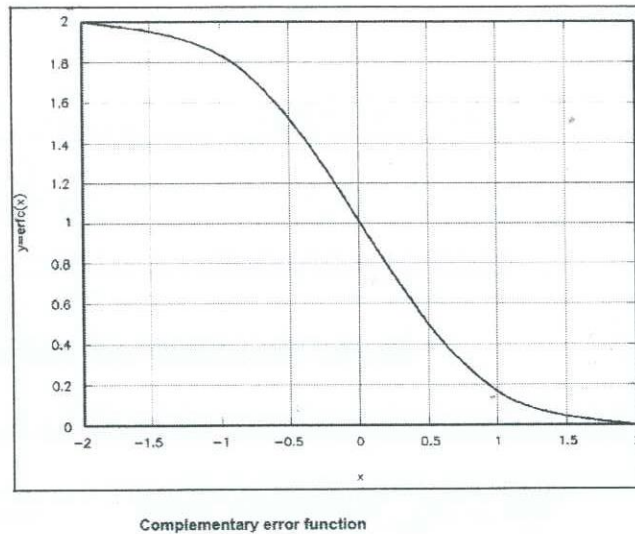


Figure Q4(b). Complementary error function.

Q5

Assume that you have been tasked to design a rock (rubble mound) breakwater consisting of an armour layer, secondary (under-layer) and a core made of quarry run.

- (a) Draw a typical cross section of a breakwater. Explain the functions of (i) armour layer (ii) secondary/under layer (iii) core (iv) footing [4 marks]
- (b) Use Van der Meer equation to calculate median rock armour diameter and mass. Assuming plunging waves, the equation reads:

$$\frac{H_s}{\Delta D_{n50}} = 6.2 S^{0.2} P^{0.18} N_z^{-0.1} \xi_{0m}^{-0.5} \text{ ----- Eq. 5.1}$$

Where, S is relative eroded area equal to 2 for initial damage. P is porosity equal to 0.3. Number of waves in a storm $N_z=5000$. Significant wave height, $H_s=3.0\text{m}$, wave period $T=8$ sec, $S_{0m}=H_s/L_0$. Breakwater slope is $\tan \alpha = 1/2$. $\Delta=[(\rho_s/\rho)-1]$ where density of rock $\rho_s=2650\text{kg/m}^3$ and density of seawater $\rho=1030\text{kg/m}^3$.

$$\xi_{0m} = \frac{\tan \alpha}{\sqrt{S_{0m}}} \text{ ----- Eq. 5.2}$$

[4 marks]

- (c) It is required to reduce wave overtopping over the breakwater. Determine the crest level of the breakwater for 2% exceedance probability (i.e., 1 in 50 year event) providing a 0.6m freeboard for settlement and water level/wave exceedance (see Figure Q5).

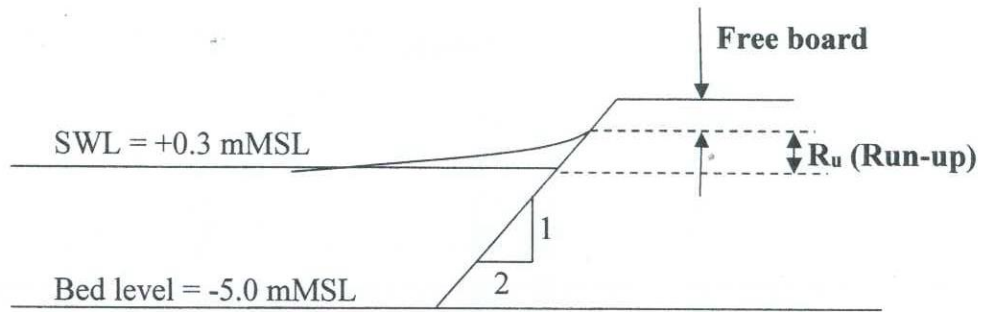


Figure Q5. Definition sketch of breakwater slope

Maximum wave run-up height (R_u) is given by:

$$\frac{R_{ui}9\%}{H_s} = A\xi_{0m} \text{ for } 1.0 < \xi_{0m} \leq 1.5 \quad \text{----- Eq. 5.3}$$

$$= B(\xi_{0m})^C \text{ for } 1.5 < \xi_{0m} \leq (D/B)^{1/C} \quad \text{----- Eq. 5.4}$$

$$= D \text{ for } (D/B)^{1/C} \leq \xi_{0m} < 7.5 \quad \text{----- Eq. 5.5}$$

$$\xi_{0m} = \frac{\tan \alpha}{\sqrt{S_{0m}}} \quad \text{----- Eq. 5.6}$$

$$S_{0m} = \frac{H_{s0}}{L_0} \quad \text{----- Eq. 5.7}$$

Assume a breakwater slope of 1:2. Design water level (SWL) is +0.3mMSL. Bed level at the toe of the structure is -5.0mMSL. Deep water wave height, $H_{s0}=2.7\text{m}$, and wave period, $T_m=8$ sec. Nearshore transformed wave height, $H_s=3.0\text{m}$. Assume a breaker index, $\gamma_b=0.78$. Table Q5 provides coefficients A,B,C and D.

[4 marks]

Table Q5. Coefficients A, B, C, D for Run-up calculation (for use in Eqs 5.3 - 5.5)

Percent exceedance	A	B	C	D
0.1	1.12	1.34	0.55	2.58
2.0	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
Significant	0.72	0.88	0.41	1.35
50 (mean)	0.47	0.60	0.34	0.82

APPENDIX:

Table 1. Wave table

h/L₀	h/L	Sinh(2πh/L)	Cosh(2πh/L)
0.030	0.07135	0.4634	1.1021
0.031	0.07260	0.4721	1.1059
0.032	0.07385	0.4808	1.1096
0.033	0.07507	0.4894	1.1133
0.034	0.07630	0.4980	1.1171
0.035	0.07748	0.5064	1.1209
0.036	0.07867	0.5147	1.1247
0.037	0.07984	0.5230	1.1285
0.038	0.08100	0.5312	1.1324
0.039	0.08215	0.5394	1.1362
0.040	.08329	0.5475	1.1401
0.041	.08442	0.5556	1.1440
0.042	.08553	0.5637	1.1479
0.043	.08664	0.5717	1.1518
0.044	.08774	0.5796	1.1558
0.060	0.1043	0.7033	1.2225
0.061	0.1053	0.7110	1.2270
0.062	0.1063	0.7187	1.2315
0.063	0.1073	0.7256	1.2355
0.064	0.1082	0.7335	1.2402
0.095	.1366	0.9677	1.3917
0.096	.1375	0.9755	1.3970
0.097	.1384	0.9832	1.4023
0.098	.1392	0.9908	1.4077
0.099	.1401	0.9985	1.4131
0.1000	.1410	1.006	1.4187
0.1010	.1419	1.014	1.4242
0.1020	.1427	1.022	1.4297
0.1030	.1436	1.030	1.4354
0.1040	.1445	1.037	1.4410

Symbols:

h = water depth, L = wave length, L_0 =deep water wave length = $gT^2/2\pi$